SOUTH ACCESS TO THE GOLDEN GATE BRIDGE
DOYLE DRIVE

DOYLE DRIVE REPLACEMENT PROJECT

♦

STRUCTURAL DESIGN CRITERIA
for
CUT-AND-COVER TUNNELS
&
NON-STANDARD RETAINING WALLS

♦

Version - September 2009

ARUP PB
joint venture

In association with Caltrans

RECORD OF REVISIONS

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0	September 2009	Final document incorporating review comments on DRAFT – Rev 4.

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1. INTRODUCTION

This document comprises the design criteria for the structural design of the cut-and-cover tunnels and non-standard retaining walls for the Doyle Drive Replacement Project, established as part of the new Presidio Parkway Alternative. The proposed replacement alignment of Doyle Drive will continue to serve as the southern approach to the Golden Gate Bridge, and will consist of a combination of at-grade roadways, bridges, and cut-and-cover tunnels.

The design criteria compiled in this document are developed as part of project-wide design requirements in partnership with California Department of Transportation (Caltrans) [1].

1.1 Structure Types

The structures covered by these criteria are the Battery Tunnels, the Main Post Tunnels, Retaining Wall No. 8, and other non-standard earth retaining structures. The tunnels are of the order of 1,000 feet long with shallow embedment.

1.1.1 Cut-and-Cover Tunnel Structures

The tunnel structure types considered include reinforced concrete cast-in-place tunnels with headwalls, U-walls, and retaining walls at the portals. Because of the relatively short lengths and shallow profiles of the Doyle Drive Project tunnel structures, "cut-and-cover", open trench, construction has been identified as the most feasible solution. After construction of the tunnel structures, soil backfill will be imported to cover the tunnels, and landscaping provided over them.

1.1.1-1 Restrictions Due to Subsurface Conditions

The existing subsurface groundwater flow across the site shall be maintained per the requirements of the project hydrological and hydraulics studies; construction excavation and shoring shall be kept to relatively shallow depths, with no deep cut-off walls that could interrupt the subsurface flow across the site. Tremie concrete slabs and/or dewatering schemes will likely be required to control groundwater flow into the excavation.

Due to the potential of liquefaction in some of the shallow underlying soil layers, the invert slabs and walls shall be of conventional cast-in-place reinforced concrete construction partially supported on either: (a) improved ground by means of cement deep soil mixing (CDSM) or installation of "full displaced rammed aggregate piers", or (b) drilled shaft and pile foundations.

1.1.1-2 Tunnel Construction Considerations

Temporary shoring systems of earth retaining walls, in conjunction with compression struts, rakers and/or tie-backs, shall retain the open cut trench vertical sides during excavation. The earth retaining systems shall be adequate to secure the retained earth and prevent any earth movements which may have detrimental effects to the nearby existing facilities. A monitoring program is required in sensitive areas to monitor ground movements during construction. In addition, tie-back walls have to meet right-of-way (ROW) easement requirements, account for the presence of existing foundations that support buildings and other facilities, and comply with sensitive constraints of the National Cemetery near the Southbound Battery Tunnel.

The shoring walls shall consist of diaphragm walls (concrete slurry or cement-soil mix), soldier pile-tremie concrete (SPTC) walls, deep soil-mix (DSM) walls, secant pile walls, sheet piles or soldier piles with wood lagging, or a combination of the above. However, groundwater flow requirements preclude the use of continuous wall systems that would cut off groundwater flow below the tunnels. Noise and vibration associated with pile driving is restricted in certain areas of the project, which will be a constraining factor. Membrane waterproofing shall be provided where the tunnel structures are exposed to groundwater. Waterproofing should envelope the entire tunnel structure (walls and roof).

1.1.2 Earth Retaining Structures

The design of the earth retaining systems shall be based on the following factors:

- The physical properties of the soil throughout and beneath the cut
- The elevation of maximum groundwater table
- The width and depth of the excavation
- The configuration of the subsurface structure to be constructed within the cut
- The size, foundation design, and proximity of adjacent structures
- Requirements for street decking across the excavation
- Traffic and construction equipment surcharge adjacent to the excavation
- Project performance requirements for different level design earthquakes (specified in Section 6.)

The determination of the earth retaining structures shall be based on the site geotechnical investigation [2], the project geotechnical report, and the recommendations of the project geotechnical engineer.

1.1.3 Tunnel Systems

Separate technical documents cover the design requirements of ancillary facilities associated with the tunnels, involving ventilation, emergency egress, mechanical/operational buildings, drainage, fire protection systems, power, lighting and traffic control systems.

1.2 General Design Requirements

The overall design requirements for the cut-and-cover tunnels and non-standard retaining walls in the Doyle Drive Replacement Project are governed by the Project-Specific Design Criteria [1]. The general design and performance requirements of the project include:

- Minimum design life of the tunnels and walls shall meet project specific criteria, but shall not be less than 75 years.
- Tunnels shall have waterproofing protection and/or dewatering pumps to prevent water leakage into the tunnels.
- Detailing for corrosion protection of reinforced concrete members and other structural members.
- Lane width, vertical clearance, shoulder widths, horizontal alignment, vertical profile, sight distance, and super elevation shall meet Caltrans standards, as modified by the project-specific criteria. Any deviation away from the Caltrans Standards shall have a design exemption.
- The seismic performance requirements outlined in Section 6 Seismic Design.
- Fire resistance of the tunnel structures to resist thermal loading (temperature gradient) resulting from the design fire established for the project.

Any design standard exceptions requires Caltrans approval.

1.3 Limits of Applicability

These design criteria only address the structural design and analysis requirements specific to cut-and-cover tunnels, non-standard earth retaining structures, and temporary shoring walls pertaining to the tunnels. Design elements not covered by this document shall be governed by project-specific structure design criteria [1] or Caltrans Standards.

2. APPLICABLE CODES AND STANDARDS

The design of the cut-and-cover tunnels and portal structures, and nonstandard earth retaining structures shall be in compliance with the Doyle Drive Project Specific-Structure Design Criteria [1].

The structural design is governed by the requirements of the Caltrans Bridge Design Specifications, using the principles of the Load and Resistance Factor Design (LRFD) method, as applicable and as modified and/or augmented herein via the referenced standards, guidelines, or technical requirements [3],[4].

Applicable provisions of the following codes, standards, and guidelines shall be considered for the preliminary and final design of the cut-and-cover tunnels and portals, and nonstandard earth retaining structures.

- Caltrans Bridge Design Specifications (BDS). LFD Version, April – 2000 [5].
- Caltrans, Seismic Design Criteria (SDC), Version 1.4 2006 [6].
- Caltrans Bridge Memo to Designers (MTD), 2004
- Caltrans, Bridge Design Aids Manual, 1995
- Caltrans, Trenching & Shoring Manual, January 1990, Revision 12, 01/00
- (AASHTO-LRFD) LRFD Bridge Design Specifications, Fourth Edition (2007) published by the American Association of State Highway and Transportation Officials with California Amendment (V.3.06.01).
- (ACI-318) Building Code Requirements for Structural Concrete and Commentary, ACI 318-05 & ACI 318R-05 published by the American Concrete Institute.
- (ACI-350) Code Requirements for Environmental Engineering Concrete Structures and Commentary, ACI 350-01 & ACI 350R-01 published by the American Concrete Institute.
- ASTM – American Society for Testing and Materials
- (AISC) Steel Construction Manual; 13th Edition (2005) published by the American Institute of Steel Construction.
- (AISC-ASD) Manual of Steel Construction Allowable Stress Design; 9th Edition (1989) published by the American Institute of Steel Construction.
- (AISC) LRFD Manual of Steel Construction, 3rd Edition (2003) published by the American Institute of Steel Construction.
- (ASCE 7) Minimum Design Loads for Building and Other Structures (2006) published by the American Society of Civil Engineers/Structural Engineering Institute.
- (AWS D1.1) Structural Welding Code – Steel D1.1/D1.1M (latest edition), published by the American Welding Society.
- (AWS D1.5) Bridge Welding Code – AASHTO D1.5M/D1.5 (latest edition), published by the American Welding Society.
- California Building Code (CBC), 2007 Edition [8].
- (PCI) PCI Bridge Design Manual, MNL-133-97, published by the Pre-stressed Concrete Institute.
- (PTM) PTI Post-Tensioning Manual, 5th Edition, November 1990, published by the Post-Tensioning Institute.
- Occupational Safety and Health Administration (OSHA) standards
- (ADA) Americans with Disabilities Act.
- 2006 Caltrans Standard Specifications

3. MATERIALS

All materials used in temporary and permanent construction for the Doyle Drive tunnel and non-standard retaining wall structures shall meet the requirements stated herein.

3.1 Concrete

Normal weight concrete, in conformance with cast-in-place and pre-cast concrete classes per Caltrans Standard Specifications shall be used. Minimum 28-day design compressive strength (f'_c) and class of concrete shall meet the requirements specified in Table 3.1 for different applications.

Table 3.1 Concrete Strength

Component	Minimum Design Strength, f'_c (psi) [1]
Cut-and-cover Tunnel	4,000
Retaining Walls	4,000
Portals and U-Sections	4,000
Buildings	4,000
Footings	4,000
Secant & Tangent Piles or Slurry Walls	4,000
Soil Cement Mix Wall	200
Mud slab	2,300 (Commercial Class)
Underwater	2,400 (Seal Concrete)
Shotcrete	4,000
Soldier Pile Concrete	4,000

1. *In certain cases, strength of concrete other than specified above may be required.*

The modulus of elasticity for design calculations of normal weight reinforced concrete shall be based on:

$$E_c = (w_c)^{1.5}\, 33\sqrt{f'_c} \quad \text{psi}$$

where:

f'_c = minimum compressive strength in psi
w_c = unit weight in lb/ft^3 (See Table 3.2)

For resistance against governing seismic demands, excluding shear, expected values of material properties shall be used. Expected values of ultimate concrete stresses, stains, and effects of confined concrete shall meet the Caltrans Seismic Design Criteria requirements and specifications.

3.2 Reinforcing Steel

All reinforcement shall be ASTM A706, Grade 60 steel, unless otherwise approved by Caltrans. Design properties shall be based on:

Modulus of Elasticity: Es = 29,000 ksi
Specified Minimum Yield Stress: f_y = 60 ksi

For resistance against governing seismic demands, expected values of material properties shall be used. Expected yield strength, strain hardening, ultimate stresses and strains of reinforcing steel shall meet the Caltrans Seismic Design Criteria requirements and specifications.

3.3 Other Materials

Pre-stressing steel, soil nails, tie-backs, rock bolts, structural steel, timber, or other materials used for design of permanent and temporary structures or during construction shall meet the AASHTO-LRFD requirements, as modified and amended by the Project-Specific Structure Design Criteria [1].

3.4 Material Density

The unit weights of materials for use in design are shown in Table 3.2

Table 3.2 Unit Weight of Materials

Material	Unit Weight **w** (lb/ft^3)
Concrete	
unreinforced	145
reinforced	150
Steel	490
Ground Water	62.4
Compacted Soil	
saturated	140
buoyant	68.0
Gravel	120

4. DESIGN LOADS AND COMBINATIONS

4.1 General Requirements

The design of the tunnel structures and non-standard retaining walls shall be consistent with Caltrans and AASHTO bridge design specifications, using the Load and Resistance Factor Design (LRFD) method. The structures shall be designed to resist load effects due to construction staging, dead weight, superimposed dead load, live load, earth pressure/surcharge, hydrostatic water pressure, thermal gradients, creep and shrinkage, wind load, and seismic load.

For the evaluation of load effects during staged excavation, effects due to soil-structure interaction, non-linear behavior, and temporary construction loads shall be considered.

For secondary structures such as light standards, signage and other external elements, wind loading criteria shall be determined per the Project-Specific Design Criteria [1].

4.2 Load Categories

Sections 4.3 and 4.4 describe the permanent and transient loading effects to be considered for the structural design of the tunnels and nonstandard retaining walls. The following symbols/abbreviations are used throughout this document for different loading sources.

Permanent Loads

CR: Creep (force effect due to)
DC: Dead Load of tunnel structural components and non-structural attachments directly supported by the cut-and-cover structure.
DD: Downdrag force
DW: Dead Load of wearing surfaces and utilities
EH: Horizontal Earth Pressure Load at final construction stage.
EL: Accumulated locked-in effects resulting from the tunnel excavation and construction sequence.
ES: Earth Surcharge Load
EV: Vertical Pressures from dead load of earth fill
SH: Shrinkage (force effect due to)

Transient Loads

CT: Vehicular Collision Force
EQ: Earthquake Effects
FR: Friction Load
FT: Thermal Gradient due to Design Fire
IM: Vehicular Dynamic Impact Load
LL: Vehicular Live Load
LS: Live Load Surcharge
PL: Pedestrian Live Load
SE: Settlement (force effect due to)
TG: Temperature Gradient (force effect due to)
TU: Uniform Temperature (force effect due to)
TS: Tsunami
WS: Wind Load on structure
WA: Water Load (Hydrostatic)

4.3 Cut-And-Cover Tunnels

The permanent and transient design load specifications, limit states, load factors, and resistance factors required for the LRFD design of the cut-and-cover tunnels are specified in Sections 4.3.1 and 4.3.2.

4.3.1 Design Loads for Cut-and-Cover Tunnel Structures

4.3.1-1 Permanent Vertical Loading (DC, EV, ES, DD)

Dead Load (DC) - The dead loads used for the design of the cut-and-cover tunnel structure shall consist of the weight of the basic structure components, the weight of elements permanently supported by the structure, including the actual weight of utilities, tunnel systems equipment, maintenance access facilities supported by tunnel components and the weight of earth cover above the tunnel roof.

The dead load shall be applied in stages to represent the construction and backfill lift sequences anticipated to construct the tunnels. The analyses shall include the maximum and minimum loadings that may be imposed on the structure either during construction or resulting from future removal of the earth cover.

The design unit weight of earth shall not be less than 140 pcf for the analysis of the structural frame, unless specified otherwise by the Geotechnical/Landscape Architectural specifications. In making calculations with regard to the dead load resisting flotation of the structure, the actual unit weight of backfill placed over the structure shall be used, but in no case shall be greater than 120 pcf.

Minimum Earth Cover for Design (EV) - The cut-and-cover tunnel and underground structures shall be designed for the actual cover depth. The minimum depth of earth cover shall be assumed to be no less than 4 feet for the tunnels at the highest point of the structure. The design check shall include worse-case conditions, such as construction stages without earth cover (i.e., uplift/flotation check).

Loads from Adjacent Buildings and Structures (ES) - Underground structures shall be designed to support lateral earth surcharge loads from adjacent buildings or other structures, including both existing structures and future construction as allowed under existing zoning and land use regulations.

Design loads on the underground structures and underpinning loads from existing structures shall be based on the actual weight and the maximum occupancy for which the building is suitable, in accordance with the original building design information and regulations, as well as current California Building Code requirements.

Loads from Existing Bridge Structures (ES) - Portions of the tunnel structures that are to be constructed adjacent to the existing bridge structures, while the route remains in service, shall be designed to support the lateral earth surcharge loading imposed by the existing bridges during construction. Design loading shall be based upon the actual dead weight of the existing structure as well as standard bridge vehicle loads.

Downdrag Loads on Piles (DD) – When pile extensions are provided to resist liquefaction induced effects and to provide support for the tunnel walls, down drag loads on the piles shall be considered and added to the dead loads on the piles.

4.3.1-2 Live Load (LL, IM, PL)

Design live load shall consist of any non-permanent load placed on or in the tunnels. Where vehicles can gain access above the tunnel and the depth of fill over the crown of the tunnel is less than 10-ft thick, the tunnel roof shall be designed for HL-93 loading, as well as a live load of 250 psf (not in combination). Vehicular live loading applied to the tunnel invert shall correspond to HL-93 loading. In addition, per California amendments to AASHTO-LRFD requirements, the route shall be designed to support P15 and P9 Permit Vehicles as well [4]

Vehicular Live Loads and Vehicular Dynamic Load (LL, IM) -, The minimum live load for the soil covered configuration of the tunnels supporting vehicular load on the roof of the tunnels shall meet the requirements for wheel load distributions to the tunnel roof structure as provided in Table 4.1.

Table 4.1 Live Load Distribution

Soil Cover Thickness	Live Load Distribution*
< 2'-0"	Distribute as a concentrated load applied directly to the top of the roof slab
2'-0" to 8'-0"	Distribute over square area sides determined as follows: W = 1.75 + 0.125 (d–2)
> 8'-0"	Distribute over square area sides determined as follows: W = 2.5 d

** W = width of square area, in feet, d = depth of soil cover, in feet*

When the distribution areas overlap, the total load shall be uniformly distributed over an area defined by the outside limits of the aggregate areas.

Impact allowances (IM) for traffic loading on the underground structures shall be provided in accordance with AASHTO-LRFD in accordance with the following formula:

$$IM = 33(1 - 0.125\,Ds) \geq 0\%$$

where: IM = Impact Allowance, %
Ds = depth of soil cover, in feet

Vehicular Live Load (LL) - The highway traffic live loading applied to the invert slab of the tunnels shall be based on AASHTO HL-93. Impact loading shall not be included for highway traffic on the tunnel invert slab. In addition, per California amendments to AASHTO-LRFD requirements, the route shall support P15 and P9 Permit Vehicles, as well.

At Halleck Street Overcrossing, the tunnel roof shall be designed for Bridge Live Load conditions, per Caltrans Bridge Design Specifications. Live loads on the tunnel roof in other areas shall consider pedestrian/public use, agency specific maintenance vehicles, and/or emergency vehicle loads as applicable.

Public Access Loading (LL) – The following public access loading conditions shall be included in the live load calculations:

- Surface streets/driveways subject to truck loading:

  AASHTO HL-93 or 250 psf (min.)
- Pedestrian/bicycle sidewalks not subject to truck loading:

  100 psf

Machine and Equipment Rooms (LL) - The live load in addition to permanent equipments and dead loads in equipment rooms shall include:

- Electrical/pump/service/machine rooms: 250 psf or actual load
- Fan rooms and battery rooms: 350 psf or actual load

Pedestrian Live Load (PL) – The live load due to pedestrian traffic and occupancy shall be based on:

- Public pedestrian areas and ramps: 100 psf
- Stairways: 100 psf or concentrated load of 300 lbs on the center of stair tread
- Maintenance walkway: 100 psf

4.3.1-3 Lateral Earth Pressure and Locked-in Loads (EH, EL)

The cut-and-cover tunnel structures shall be designed for lateral soil pressure imposed by the earth abutting against the structure (EH).

The tunnel structures shall be designed for side-sway loads resulting from unbalanced lateral earth pressures caused by (short-term) construction staging and (long-term) permanent configurations, such as in the case of the Main Post tunnels where the soil cover over the tunnel's north face forms an embankment.

Soil pressure diagrams shall be developed by the geotechnical engineer based on site-specific and stage-specific conditions. To capture the influence of the rigid diaphragm walls (e.g., secant pile walls and slurry walls) on earth pressures and forces imposed on the structure, and to account for the non-linear behavior of the tunnel structure during tunnel excavation, soil-structure numerical modeling analysis methods may be used (finite element or finite difference).

Lateral earth pressures imposed by surcharge loading during construction (EL) shall also be considered.

4.3.1-4 Hydrostatic Pressure (WA)

Site specific ground water conditions and flow shall be considered to determine hydrostatic pressure loading components on the tunnel/retaining walls and slabs. The effects shall consider the recorded as well as pending site-specific investigation of subsurface water flow. Effects of water passages, seepage, or asymmetric water pressure on the side walls of the tunnel, if present, shall be provided and considered by the design.

Buoyancy forces shall be computed at 62.4 psf / ft of depth below the design groundwater table. The groundwater levels shall be based on the project final Geotechnical Report.

Adequate resistance to flotation shall be provided at all sections for full uplift pressure on the structure foundation, based upon the maximum probable height of the water table. For the completed structure, such resistance shall consist of the dead weight of the completed structure plus the weight of backfill overlying the structure within vertical planes drawn through the outer edges of the structure, and through all joints separating adjacent tunnel sections. It shall be assumed that the construction sequence will maintain dewatering operations in place until the tunnel structure is completed and backfill is in place.

When evaluating buoyancy loads, the compacted dry weight of soil above the roof level shall be used, assumed at no more than 120 pcf. The weight of street pavement, surface improvements, interior finishes, equipment, side wall friction, and live loads shall be neglected for the purposes of calculating uplift resistance.

Uplift effects shall be checked for 100-year and 500-year flood events using the redundancy factors, η_R, given in Table 4.2 in calculating the factored loads including backfill in place and before completion of interior finish; skin friction effects shall be neglected in computing the factored resistance (see Section 4.3.2).

Table 4.2 Redundancy Factors for Tunnel Hydrostatic Uplift

Flood Event (Return Period, years)	Redundancy Factor η_R * (Cut-and-cover Tunnels)
100	1.10
500	1.05

** Exclude any resistance by soil-wall frictional resistance.*

4.3.1-5 Earthquake Loads (EQ,TS)

The earthquake loading (EQ) criteria for the design of the tunnel structures shall be based on a site-specific seismic hazard assessment (SHA), which establishes design acceleration response spectra (ARS) representative of rock-outcrop and ground surface motions. The requirements are discussed in Section 6.

Tsunami loading (TS) conditions shall be considered as part of the seismic hazard, as they are associated with a much higher mean return period compared to conventional flood loading. These load effects shall be based on project specific hydraulic criteria. The tsunami load case shall be included for structural design resistance, as well as protection against public flooding within the tunnels. Buoyancy effects caused by tsunami loading shall be included as hydrostatic and buoyancy load components as an extreme limit state.

4.3.1-6 Thermal, Shrinkage, Creep, and Settlement Effects (TU/TG, SH, CR, SE)

Expansion and contraction stresses and strains due to thermal variations, shrinkage (SH), creep (CR), and differential settlement (SE) shall be considered as per Caltrans and AASHTO LRFD requirements for concrete structures. Uniform temperature (TU) variations causing stresses and strains in the longitudinal directions, as well as through-the-thickness thermal gradients (TG) causing flexure stresses and strains shall be considered and based on construction and site conditions. Recorded average temperatures at the project site range from 49.3 to 62.1^o F. The design shall be based on the Caltrans "Mild Coastal" air temperature specification for the San Francisco Bay Area. Thermal gradients across the thickness of walls and slabs are given in Table 4.3.

Table 4.3 Thermal Gradients

	Thermal Gradient, TG (^{o} F)		
Location	**Construction Phase**	**Summer**	**Winter**
Outer Surface	60	60	50
Middle Surface	60	70	45
Inner Surface	60	80	40

4.3.1-1 Design Fire Thermal Load Effect (FT)

The effect of extreme thermal loading on the tunnel structures due to the design fire shall be analyzed. The thermal stress analyses shall be performed for a typical cross section of the tunnels subjected to a steady-state thermal gradient through the thickness of the tunnel walls, invert slab, and roof slab structures as determined by the tunnels systems design criteria. The analyses shall be conducted to evaluate the stability of the tunnel structure.

4.3.1-2 Live Load Surcharge (LS)

Highway live load surcharge, as well as surface live loads imposed by heavy equipment, cranes, or other special concentrated surface loads shall be determined by analytical methods and shall consider the depth of soil cover, the distribution, extent, and intensity of the loading, and the soil type and width of the underground structure, as well as the intended/permitted construction staging and methods. The loading shall also consider construction methods and future permitted and probable loading conditions as well. The conditions shall consider intermediate unbalanced earth pressure phases due to construction staging. Staged-excavation analyses shall be performed to determine the maximum forces, stresses, and deformations occurring during tunnel excavation and construction stages.

Maximum construction equipment loads shall be based on the assumed and permitted construction methods and stage construction, and shall be identified on design documents. It is assumed that construction contractors shall provide additional temporary supports and shoring for additional construction equipments and/or methods beyond the reported design assumptions.

4.3.1-3 Friction and Wind on Structure (FR, WS)

Forces due to friction (FR) shall be based on the extreme values of friction coefficient between the sliding surfaces in contact with each other.

Wind force effects on exposed parts of the tunnel structures (WS) shall be based on dynamic pressure calculated using wind velocity profiles appropriate for the site, per the Caltrans BDS and AASHTO LRFD requirements.

4.3.2 Limit States, Load Factors & Combinations, and Resistance Factors

The tunnel structure, including all component members and sub-systems, shall be designed to resist the factored loads set forth in this section. In selecting the critical loading combinations, consideration shall be given to appropriate combinations of maximum and minimum load effects, either vertical or horizontal, which result in the largest stresses in the structure and to the unbalanced loading presented by asymmetrical conditions.

The total computed force effect, ***Q***, shall be bounded by the factored resistance as follows:

$$\mathbf{Q} = \sum_{\mathbf{i}} \eta_i \gamma_i \mathbf{Q}_{\mathbf{i}} \leq \phi \mathbf{R}_{\mathbf{n}}$$

where,

$\eta_i = \eta_i(\eta_D, \eta_R, \eta_I)$ = load modifier, function of Ductility, Redundancy, Importance
γ_i = load factor
Q_i = force effect
R_n = nominal resistance
ϕ = resistance factor

4.3.2-1 Limit States

The tunnel structure design shall be evaluated for *Service*, *Strength*, and *Extreme* Limit States, in accordance with the AASHTO-LRFD Standard. The definition of *limit state*, as provided in the AASHTO LRFD Standard, is: *the condition beyond which the structure or component ceases to satisfy the provisions for which it was designed.*

Extreme limit states for seismic loading include the Functional Evaluation Earthquake (FEE) and Safety Evaluation Earthquake (SEE) as described under Seismic Design Requirements (Section 5).

The distribution of internal forces in the structures, under the permanent and transient loads, and the capacity of the structure shall be based on the analysis methods specified in Chapter 5.

4.3.2-2 Load Factors

Load factors for design of the cut-and-cover tunnels shall be in accordance with the AASHTO-LRFD Article 3.4.1. The load combinations relevant to the Service, Strength, and Extreme limit states, with the applicable load factors, shall be based on Tables 4.4, 4.5a, and 4.5b.

Table 4.4 LRFD Limit State Load Factors

Load Combination Limit State	DC DD DW EH EV ES EL CR SH	LL IM PL LS	WA	WS	FR	TU	TG	SE	Use One at A Time			
									EQ	TS	CT	TF
Strength – I	γ_p	1.75	1.0	-	1.0	0.5/ 1.2	γ_{TG}	γ_{SE}				
Strength – II	γ_p	1.35	1.0	-	1.0	0.5/ 1.2	γ_{TG}	γ_{SE}				
Strength – III	γ_p	-	1.0	1.4	1.0	0.5/ 1.2	γ_{TG}	γ_{SE}				
Strength – IV	γ_p	-	1.0	-	1.0	0.5/ 1.2	-	-				
Strength – V	γ_p	1.35	1.0	0.4	1.0	0.5/ 1.2	γ_{TG}	γ_{SE}				
Extreme – Ia (FEE)	γ_p	γ_{EQ}	1.0	-	1.0	-	-	-	1.0			
Extreme – Ib (SEE)	γ_p	γ_{EQ}	1.0	-	1.0	-	-	-	1.0			
Extreme – II	γ_p	0.5-	1.0	-	1.0	-	-	-			1.0	
Extreme - III	γ_p	-	-	-	1.0	-	-	-		1.0		
Extreme – IV	γ_p	-	1.0	-	1.0	-	-	-				1.0
Service – I	1.0	1.0	1.0	0.3	1.0	1.0/ 1.2	γ_{TG}	γ_{SE}				
Service – II	1.0	1.3	1.0	-	1.0	1.0/ 1.2	-	-				
Service – III	1.0	0.8	1.0	-	1.0	1.0/ 1.2	γ_{TG}	γ_{SE}				
Service – IV	1.0	-	1.0	0.7	1.0	1.0/ 1.2	-	1.0				

Table 4.5a LRFD Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		γ_p Load Factor Maximum	γ_p Load Factor Minimum
DC: Dead Load of Tunnel Structure Excluding Interior Finishes & Non-Structural Elements		1.25	0.9
DC: Dead Load of Interior Finishes & Non-Structural Elements		1.50	0.9
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• AEP for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts		1.50	0.90
ES: Earth Surcharge		1.50	0.75

Table 4.5b LRFD Load Factors for Transient Loads

Load Category	Load Factor	Limit State
EQ	$\gamma_{EQ} = 0.5$	Extreme Ia and Ib
TG	$\gamma_{TG} = 0.0$ $\gamma_{TG} = 0.5$ $\gamma_{EQ} = 1.0$	Strength & Extreme Service w/ LL Service w/o LL
SE	$\gamma_{SE} = 1.0$	Strength, Extreme, & Service

4.3.2-3 Resistance Factors

Resistance factors for the design of cut-and-cover tunnels shall be in accordance with the AASHTO-LRFD Article 5.5.4.2.1. The structure resistance factors are summarized in Table 4.6. The soil resistance factors shall be obtained from AASHTO-LRFD Article 10.5.5.2.

Table 4.6 Resistance Factors, ϕ – Structures

Section Description	Resistance Factor ϕ
Tension-controlled Reinforced Concrete	0.90
Tension-controlled Pre-stressed Concrete	1.00
Shear & Torsion: • Normal Weight Concrete • Light Weight Concrete	 0.85 0.70
Compression-controlled Sections with Spirals or Ties for Seismic Zones at the Extreme Event Limit State	0.75
Bearing on Concrete	0.70
Compression in Strut-and-Tie Models	0.70

4.4 Non-Standard Retaining Walls

The permanent and transient design load specifications, limit states, load factors, and resistance factors required by the LRFD design of the non-standard retaining walls are specified in Sections 4.4.1 and 4.4.2.

4.4.1 Design Loads for Non-Standard Retaining Walls

4.4.1-1 Permanent Vertical Loading (DC, EV, ES, DD)

The dead loads used for the design of the nonstandard retaining wall shall consist of the weight of the structure components, the weight of nonstructural attachments (DC), and the weight of earth cover above the footing (EV).

<u>Loads from Adjacent Buildings and Structures, and Earth Surcharge (ES)</u> – Retaining walls shall be designed to resist loads from adjacent structures, including both existing buildings and future constructions, as allowed under existing zoning and land use regulations.

Design loads from existing buildings shall be based upon the actual weight and the maximum occupancy limit that is allowed for the building, original building design information and regulations, as well as current California Building Code requirements [8].

Design loads shall be determined for future buildings or structures based upon the zoning regulations, including current Presidio development plans. Loads shall be determined assuming future building or structures with the heaviest occupancy, height limit, and set backs as stipulated within these zoning regulations and development plans.

<u>Downdrag Loads on Piles (DD)</u> – When pile extensions are provided to resist liquefaction induced effects and provide support for the walls, down drag loads on the piles shall be added to dead loads on the piles.

4.4.1-2 Horizontal Earth Pressure (EH)

Horizontal Earth Pressure shall consist of the lateral soil pressure imposed by the earth abutting the structure and the lateral earth pressures imposed by surcharge loading, as described above.

4.4.1-3 Hydrostatic Pressure (WA)

Site specific ground water conditions shall be considered to determine hydrostatic pressure loading components on the nonstandard retaining walls, consistent with design drainage conditions, as applicable.

4.4.1-4 Live Load Surcharge (LS)

Where live load surcharge is applicable, the surcharge load shall be included over the backfill immediately above the wall for evaluation of foundation bearing resistance and structural component design. However, the live load surcharge shall not be included over the backfill for evaluation of eccentricity, sliding or other failure mechanisms for which such surcharge would represent added resistance to failure.

Live loads surcharge imposed by local street traffic, heavy construction equipment, or other special concentrated surface loads shall be determined by analytical methods with consideration of the depth of soil cover, the distribution, extent, and intensity of the loading, and the soil type behind the retaining wall, as well as the anticipated wall deflection under live load surcharge.

4.4.1-5 Vehicle Collision Load (CT)

For the Portals and U-Sections or earth retaining walls next to vehicular traffic, the requirements within AASHTO Section 3.6.5 shall apply for the protection and design of the structures for vehicular collision. This load occurs occasionally and shall be analyzed as an LRFD Extreme Event Loading with a reduced factor of safety. Particular attention shall be directed to the post-collision performance of retaining walls which support adjacent occupied buildings and/or facilities.

4.4.1-6 Earthquake Loads (EQ,TS)

The seismic loading (EQ) for the design of the non-standard retaining walls shall be as prescribed in Section 6.

As in the case of the tunnels structures, stream-like effects caused by the tsunami load case (TS) shall also be considered in design, similar to the hydrostatic and stream flow load case.

4.4.2 Limit States, Load Factors & Combinations, and Resistance Factors

The non-standard retaining wall structures shall be designed to resist the factored loads set forth in this section. The load category designations are identical to those for the tunnel structures. In selecting the critical loading combinations, consideration shall be given to appropriate combinations of maximum and minimum load effects, either vertical or horizontal, which result in the largest stresses in the structure and to the unbalanced loading presented by asymmetrical conditions.

The total computed force effect, $\boldsymbol{Q}$, shall be bounded by the factored resistance as specified in Section 4.3.2.

4.4.3 Limit States

Nonstandard retaining walls shall be investigated at the Service Limit State Load Combination I, and Strength Load Combinations I and IV, and Extreme Event Limit State Combination I and Combination II, in accordance with Caltrans-BDS and AASHTO-LRFD requirements.

4.4.3-1 Load and Resistance Factors

Load and resistance factors for the design of nonstandard retaining walls shall be in accordance with the provisions of Caltrans-BDS and AASHTO-LRFD Standard.

Load combinations relevant to the Service, Strength, and Extreme limit states, with the applicable load factors, shall be based on Tables 4.7 & 4.8

Table 4.7 LRFD Limit State Load Combinations

Limit State	DC DW DD EH EV ES	LL LS	WA	Use These One at A Time		
				EQ	TS	CT
Service – I	1.0	1.0	1.0			
Strength – I	γ_p	1.75	1.0			
Strength – IV	γ_p		0.5			
Extreme – Ia, Ib	1.0		1.0	1.0		
Extreme – II	1.0	0.5	1.0			1.0
Extreme – III	1.0	-	-		1.0	

Table 4.8 LRFD Load Factors for Permanent Loads

Type of Load	Maximum Load Factor, γ_p	Minimum Load Factor, γ_p
DC DC Strength IV, only DD	1.25 1.50 (AASHTO-LRFD Table 3.4.1-2)	0.90 0.90 (AASHTO-LRFD Table 3.4.1-2)
DW	1.50	0.65
EH Active At-rest AEP for anchored walls	 1.50 1.35 1.35	 0.90 0.90 N/A
EV Overall Stability Retaining Walls	 1.35 1.35	 N/A 1.00
ES	1.50	0.75

Resistance factors for the design of the non-standard retaining walls shall be in accordance with the requirements specified in Section 4.3.2-3, except that the resistance factor for the Extreme-I limit state can be taken as 1.0.

4.4.3-2 Stability

The stability of the non-standard retaining walls shall be ensured in the design for overall (global) stability, and resistance against overturning and sliding, in accordance to the provisions of the AASHTO LRFD Standard.

Overall stability shall be evaluated using limit equilibrium methods of analysis. Overturning effects shall be evaluated based on the allowable eccentricity of loading in the strength limit state. Resistance to failure by sliding shall be evaluated using the summation of: factored resistance of side shear between soil and foundation, and passive resistance of the soil.

5. STRUCTURAL ANALYSES

The structural design of the cut-and-cover tunnels and non-standard retaining walls shall be based on static and dynamic structural analyses using local, sub-system, and global models.

Static analyses are required for: (a) member or sub-system capacity evaluations, (b) service load evaluations, and (c) construction staging static analyses, as needed. Dynamic analyses are required for the assessment of seismic demand loads, deformations, and displacements. The structural analyses shall consider all load categories specified in Section 4, and the expected behavior of the components under the specific loading conditions.

5.1 Cut-and-Cover Tunnel Structures

5.1.1 Soil Medium Capacity and Stiffness Evaluations

The capacity and stiffness of the site-specific soil-medium supporting and surrounding the cut-and-cover tunnels shall be provided by the geotechnical engineer. Inelastic load-deflection curves characterizing discrete soil springs shall be developed by the geotechnical engineer for use as soil model parameters for use in the structural analyses for service load evaluations and soil-structure system capacity evaluations.

The evaluation of improved ground by means of cement deep soil mixing (CDSM) or installation of “full displaced rammed aggregate piers” shall be accomplished by means of numerical analyses using appropriate computer analysis codes (such as PLAXIS or FLAC). The analyses shall be used to assess the settlement potential of the improved ground under long term loading conditions.

5.1.2 Service Load Analyses

Service load analyses of the cut-and-cover structure shall be performed to determine the magnitude and distribution of all internal loads and load effects in the structural system, subject to the external design service load categories corresponding to the limit states cited in Section 4.

The analyses shall be performed by means of finite-element computer analysis codes, using local, sub-system, or global models. The tunnel liner discretization may consist of shell/plate elements or beam-column elements, and the soil medium representation may be by a series of discrete spring elements oriented normal and tangential to the structural members on the soil-structure interface. Linear-elastic properties shall be used for the tunnel liner. The springs representing the surrounding soil shall be based on site-specific geotechnical properties, considering nonlinear stress-strain relations, as cited in Section 5.1.1. Elasto-plastic or effective linearized springs, obtained by iteration, may be used as applicable, as well as inelastic properties for the soils.

Where a waterproofing membrane is used between the exterior of the tunnel and surrounding material, tangential soil springs may be ignored. In this type of model, loads and/or displacements are predetermined and applied directly to the walls through the soil springs.

5.1.3 Structural Capacity Evaluations

The capacity analyses of the cut-and-cover tunnel structures shall be performed by means of finite-element computer analysis codes, using local or sub-systems models with linear and non-linear models for the structural members and surrounding soil medium, as described in Section 5.1.2 for the service load analyses. Appropriate manual analytic techniques may also be used for the determination of ductilities of individual flexural members.

Sub-system models shall be utilized for ductility capacity evaluations of the tunnel structure cross sections, by means of displacement-based push-over load cases that simulate transverse racking deformations of the soil-structure system. Load-deflection curves (tunnel wall base shear vs. racking drift) of typical tunnel cross sections shall be developed for evaluating the seismic performance of the structure.

3D global models and analyses may be required to asses the capacity of the tunnel structures to resist differential ground settlement.

5.1.3-1 Moment-Curvature Analysis

Moment-curvature (M-ϕ) analyses shall be performed to determine the flexural capacities of unit slices of the tunnel walls, roof, and invert slabs sections. (M-ϕ) analyses of the tunnel cross-sections themselves may also be performed to estimate the overall flexural capacities of the tunnel structure.

Analytical plastic hinge length (L_p, extent of distributed plasticity) needed to determine the spread of plastic curvature in flexural members, shall be evaluated based on the Caltrans procedure for bridge columns, and shall account for component thickness, aspect ratios, rebar size, and confinement effects The same procedure shall be applied to strips of tunnel cross section wall units and slab elements.

5.1.3-2 Soil-Structure Interface

The modeling of the soil-structure interface of the tunnels shall be performed based on the following observations:

- If *soil-gapping* is determined to be important by the geotechnical engineer, appropriate non-linear *contact-gap* elements should be used to account for this component at the interface.
- The tangential interface boundary conditions (full-slip or no-slip) between the tunnel liner and the surrounding soil shall be evaluated if it is deemed to be an important factor by the geotechnical engineer. The interface shall be modeled by using appropriate tangential springs that allow slippage when the side shear capacity of the soil is exceeded at the interface.

5.1.4 Seismic Analyses

The design of the cut-and-cover tunnels shall be evaluated for its ability to resist earthquake ground motions, as defined by the site-specific seismic hazard assessment presented in Section 6. The seismic evaluation shall be performed by the following procedure:

1. Establish site-specific design ground motions.
2. Perform site-response analyses of representative soil profiles along the tunnel alignment.
3. Perform racking analyses of typical tunnel cross-sections based on simplified analytic techniques.
4. Perform dynamic soil-structure interaction (SSI) analyses

The requirements of these analyses are presented below.

5.1.4-1 Site Response Analyses

Site response analyses of representative soil profiles shall be developed to determine the free-field motions and strain-compatible profiles, using 1D wave propagation utilizing the computer analysis codes such as SHAKE [9]. The locations of the representative soil profiles shall be determined by the geotechnical engineer.

For horizontal seismic loading induced by vertically propagating shear-waves (SV- and SH-wave fields), small-strain and strain-compatible soil properties and model parameters comprising unit weight, shear-wave velocity or shear modulus, and damping profiles. For vertical seismic loading, induced by vertically propagating compression waves (P-wave field), the effect of the water table shall be taken into account in developing the small-strain soil profile.

5.1.4-2 Transverse Racking Analyses

Transverse racking analyses of typical cut-and-cover tunnel cross-sections shall be performed to obtain an initial evaluation. The pseudo-static analysis methods proposed by Penzien [11] shall be used for the analyses, utilizing two-dimensional (2D) models consistent with the capacity analysis models.

5.1.4-3 Dynamic Soil-Structure Interaction

Dynamic soil-structure interaction (SSI) analyses shall be performed to evaluate the seismic performance of the tunnel structures, using computer analyses codes with the capability of capturing SSI effects (such as SASSI).

The SSI analyses shall be used to evaluate the overall dynamic response of the soil-structure system, in terms of overall displacements, transverse racking drift deformations of the tunnel cross-section, longitudinal dilatations, and curvature deformations (bending about the vertical and transverse axes of the tunnel alignment). As a minimum, two-dimensional (2D) models shall be utilized to assess the racking demand on the typical tunnel cross-sections. Three-dimensional (3D) models will be required to assess the effects discussed in sub-section 5.1.4-4.

Appropriate depth-variable representations of the soil medium and the associated free-field motions obtained from site-response analyses of representative soil profiles along the tunnel alignment shall be used in the SSI analyses. In order to capture non-linear characteristics of the soil profiles under consideration, strain-compatible model parameters of the soil layers (shear modulus, G, or shear wave velocity, Vs, and damping, ζ, profiles) shall be used as discussed in Section 5.1.4-1. When using linear-elastic flexural elements for the structure, appropriate reductions in element stiffness due to concrete cracking shall be accounted for based on the cross-section moment-curvature analyses.

The SSI analyses shall consider vertically propagating shear wave fields (SV- and SH-wave) and a compression wave field (P-wave) to simulate the three orthogonal seismic loading components: 2 horizontal and 1 vertical. Directional load combinations resulting from the three components of the earthquake loading shall be performed by the simultaneous application of the three seismic load component. In the case of linear-elastic analyses (using equivalent-linearized model parameters), the response quantities of interest may be obtained by direct superposition of results obtained for each load component separately.

5.1.4-4 Three Dimensional (3D) Effects

Where significant variations in either tunnel cross-section or geological conditions occur along the tunnel alignments, three dimensional (3D) models may be required to capture the response of the tunnel along three orthogonal axes. 3D models would be required to capture the following effects:

- Transitions at portals
- Transitions to differing tunnel cross sections
- Intersections of cross passages, if implemented
- Changes in ground motion due to both wave propagation and attenuation effects that result in adverse tunnel movement
- Change due to differing ground conditions
- Liquefaction and lateral spread effects
- Changes in slope in a longitudinal direction to the tunnel

A 3D global analysis model shall be used to capture the three primary modes of deformation that occur under seismic ground shaking: transverse racking, axial compression/extension, and curvature (i.e., flexural bending) deformations.

5.1.4-5 Ground Water Effects under Dynamic Loading Condition

For the tunnel structures surrounded by saturated course granular soils (such as course sand and gravel), the hydrodynamic effect of the ground water should also be taken into consideration under the seismic loading condition.

This effect can be assessed by using the Westergaard theory (originally derived to obtain the dynamic water pressure on the face of a concrete dam). In deriving the hydrodynamic water pressure, the horizontal seismic coefficient used should be consistent with the design peak ground acceleration at the site. In addition, to account for the interaction between the course granular soil and the water, the total hydrodynamic force can be reduced to about 70% of that calculated from the original Westergaard theory.

5.1.4-6 Liquefaction and Liquefaction-Induced Ground Deformations

The effects of liquefaction and liquefaction-induced ground deformations shall be evaluated at relevant locations along the tunnel alignment, including in the areas where potential slope instability and landslides that could potentially damage the tunnel structures. The adverse effects that need to be considered include the following:

- Uplift, buoyancy, and flotation of the tunnels;
- Post-liquefaction settlements and deformations (total as well as differential);
- Lateral sliding stability of the tunnel;
- Loss of bearing capacity, if applicable;
- Down drag and reduction in lateral/vertical resistance of deep foundations, if applicable; and

An initial screening study, such as along the lines of NCEER [12] should be conducted, and then followed by more refined analyses and evaluation of the impact on the tunnel structures. If the liquefaction impact analyses indicate a potential for rendering the structures unsuitable for their purpose owing to movement, appropriate mitigation measures shall be incorporated into the design.

Empirical procedures based on SPT (standard penetration test) blow counts or CPT (cone penetrometer testing) data shall be used for liquefaction potential evaluations [13].

Post-liquefaction settlements occurring as excess pore-water pressure generated during the earthquake that gradually dissipate after the shaking ends shall be estimated by multiplying the post-liquefaction volumetric strain by the thickness of the liquefiable layer based on the average cyclic shear stress induced by the earthquake and the penetration resistance of the soil. The post-liquefaction volumetric strain can be estimated using either the procedure developed by Tokimatsu and Seed [14] or the empirical chart developed by Ishihara and Yoshimine [15].

5.1.5 Structural Analyses of Tunnel Structures for Extreme Thermal Loading (Design Fire)

Structural analyses shall be performed to evaluate the effect of extreme thermal loading on the tunnel structures due to the design fire established for the project. The thermal stress analyses shall be performed for a typical cross section of the embedded tunnels subjected to a steady-state thermal gradient through the thickness of the tunnel walls, invert slab, and roof slab structures as determined by the tunnels systems design criteria.

The analyses shall be conducted to evaluate the stability of the structure (roof supporting soil cover), using finite-element computer analysis software (such as ADINA) with capabilities to model temperature-dependent material properties and non-linear constitutive behavior.

5.2 Non-Standard Retaining Walls

5.2.1 Demand Evaluations

Structural analyses of the non-standard retaining walls shall be performed using appropriate analytic methods or finite-element software to estimate the distribution of loads and deflections of the wall.

The seismic demand analyses for non-standard yielding retaining walls (i.e., non-rigid structure allowed to deflect sufficiently to mobilize active pressure behind the structure), non-gravity cantilevered walls and anchored walls (as defined in AASHTO-LRFD Section 11) shall be performed based on flexural (local) models subject to seismic pressure in addition to static pressure computed from ground acceleration. The seismic pressures may be estimated based on the Mononobe-Okabe method.

For rigid (i.e., non-yielding) non-standard walls, the seismic loading may be computed utilizing the racking analysis procedure based on free-field shield strain (see Section 5.1.4-2).

Dynamic SSI analyses may also be used to estimate the seismic loading coefficients of the wall. The site-specific ground motions, soil conditions, and wave propagation effects shall be the basis of defining seismic coefficients and yield accelerations for the SSI analysis of the wall structure.

5.2.2 Capacity Evaluations

Consistent with the project performance goals (such as serviceability and reparability, or no-collapse), material strain limits shall be established for the wall structures, with well detailed plastic hinge locations, as applicable to the wall system. Similarly, limits of deformations and movements shall be examined by local analytic models in which nonlinear performance of ductile components are taken into account.

Displacement and deflection limits of the walls shall be established based on recommendations of the final geotechnical report and evaluation of their impact on adjacent buildings and facilities supported on the retained earth.

6. SEISMIC DESIGN

The seismic design of the structures comprising the Doyle Drive alignment shall be based on the performance-based criteria established by the Caltrans-SDC, AASHTO-LRFD Standard, and Project-Specific Structure Design Criteria, as amended by the provisions of this chapter.

6.1 General Performance Requirements

The Doyle Drive alignment is categorized as a 'Recovery Route' as designated by Caltrans. In coordination with Caltrans, the seismic design of the tunnel structures and the non-standard retaining walls shall be performed using site-specific ground motions for two performance levels; Safety Performance Level and Functionality Performance Level.

The performance-based seismic design criteria shall conform to the deformation-based design approach, based on the analysis methods cited in Chapter 5. The deformation-based performance measures shall be the basis of seismic design, using the appropriate target ductilities and corresponding strain limits established in this chapter. The general seismic performance parameters that govern the design of this 'Recovery Route' are as follows:

- *Serviceable* performance after a seismic event requires immediate full traffic access after a short period of inspection or minor repairs. A maximum delay of 72 hours is permitted. See FEE performance level in Table 6.1.
- *Repairable* performance after a seismic event requires limited immediate access to emergency vehicles, with only repairable damage. Upon repair, the facility shall support full traffic access. "Repairable Damage" can be defined as allowing moderate inelastic response of the lining and portals to occur. Concrete cracking, reinforcement yield, and spalling of cover concrete is expected at this level of inelastic response. The extent of damage should be sufficiently limited to permit restoration of the structure to essentially the pre-earthquake condition without replacement of any portion of the lining or portals. See FEE performance level in Table 6.1.
- *No-Collapse* performance after the seismic event requires structure stability for public safety in accordance with ductility demand and capacity values documented in the SDC. See SEE performance level in Table 6.1.

6.1.1 Seismic Performance Levels

The seismic design of the tunnel structures shall be based on two performance levels: *Safety* and *Functionality,* corresponding to 'Upper' and 'Lower' level earthquake events designated: Safety Evaluation Earthquake (SEE), and Functionality Evaluation Earthquake (FEE), respectively. Table 6.1 summarizes the performance requirements under the dual-level seismic loading criteria.

Table 6.1 Minimum Seismic Performance Levels

Design Earthquake	Performance Level
Functionality Evaluation Earthquake (FEE)	**Functionality Performance Level** • Repairable-to-Serviceable damage, with or without traffic restrictions • Immediate access to emergency vehicles following inspection
Safety Evaluation Earthquake (SEE)	**Safety Performance Level** • Significant damage / No-Collapse: life safety assured • Limited Service

Per the guidelines adopted by Caltrans and the return period risk determined for the project, site-specific hazard analyses shall be performed to establish the design response spectra and ground motions for the SEE and FEE as follows:

- **Safety Evaluation Earthquake (SEE) –** The *upper level* event to be used for the design shall be based on the acceleration response spectrum (ARS) derived from the envelope of the median (50th percentile) deterministic Maximum Credible Earthquake (MCE) ARS and a probabilistic hazard ARS for an event with a mean return period of 1,000 years (i.e., 5% probability of exceedance in 50 years).

- **Functionality Evaluation Earthquake (FEE)** – The *lower level* event to be used for the design shall be based on a probabilistic hazard ARS for an event with a mean return period of 108 years (i.e., 50% probability of exceedance in 75 years).

Seismic performance requirements of structures under construction and temporary shoring structures shall meet the Caltrans requirements for temporary bridges or bridges under temporary conditions carrying public vehicular traffic [Caltrans Memo-to-Designers 20-12]. The seismic loading for such structures shall be designated as the **Construction Evaluation Earthquake (CEE)**, which shall be equal to the acceleration response spectrum developed for the FEE.

6.1.2 Component and Sub-System Performance Criteria

The performance-based design requirements on the primary components of the lateral and load resisting systems of the tunnel structures are categorized in Table 6.2, in terms of displacement ductility and capacity-protected designations.

The structural design of the tunnels shall consider limiting the seismic demand on critical components under high combined axial, shear, and bending loads (such as roof and invert slabs), employing the principles of capacity-protection design, where plastic hinging is confined to predetermined locations and components in the load path. Overstrength factors to be applied to the controlling capacities of the capacity-protected members shall be 1.2 for capacities based on ultimate strain, and 1.5 for capacities based on nominal strain.

Inelastic response shall be limited to specific flexural components (with circular solid confined cores using hoops) and special lateral load resisting sub-systems which have been detailed to accommodate ductile plastic hinging. The design of flexural members shall ensure that the shear capacities of the members are greater than the plastic shears corresponding to the associated plastic moments at the member extremities to ensure formation of ductile plastic hinges.

Table 6.2 Component Performance Category

Component	Component Performance Criteria			
	Functional Evaluation Earthquake (FEE)		Safety Evaluation Earthquake (SEE)	
	Ductile [1]	Capacity-Protected [2]	Ductile [1]	Capacity-Protected [2]
Tunnel Roof Slab		X		X
Tunnel Invert Slab		X		X
Tunnel Walls	X		X	
Retaining Walls	X		X	
Footings		X	X	
Piles		X	X	
Columns	X		X	
Bulkheads		X	X	

1. *Deformation based measure, of inelastic behavior, evaluated using strain-based section analyses.*
2. *D/C ≤ 1.0 for protected components compare component demands from governing seismic analyses including plastic hinging of ductile components and soil structure interaction. Overstrength flexure strength of adjacent ductile components if applicable (per Caltrans requirements) shall be used for demand (D). Component Capacity (C) refers to nominal component strength.*

6.2 Ground Motions

The earthquake loading criteria for the design of the tunnel structures and non-standard retaining walls shall be based on a site-specific seismic hazard assessment (SHA), which is required to establish design acceleration response spectra (ARS) representative of rock-outcrop and ground surface motions at the project site.

In the development of the SHA, benchmark criteria shall be established for conditions that are characteristic of the selected rock attenuation relationships. The Next Generation Attenuation (NGA) model shall also be considered in developing the site-specific ARS, based on appropriate averaged shear wave velocities (V_{s30m}) of the upper 30 meters of the local geology.

Ground motions representative of the SEE and FEE design events will be required for the dynamic analyses of the tunnels and non-standard retaining wall structures. Spectrum-compatible motions, with the design ARS as target spectra, shall be developed to represent the Fault-Normal (FN), Fault-Parallel (FP), and Vertical (V) components of the ground motions at the site.

For the purposes of variability, a minimum of three sets of motions shall be developed, for both the SEE and FEE events.

Standard practice shall be followed to produce site-specific, spectrum-compatible, ground motions, using recorded ground motions of representative earthquakes as 'seed' motions for the synthesis process. The significance of the following factors shall be evaluated and accounted for in the development of the ground motions: (a) Near-fault Rupture Directivity associated with the FN component; and (b) Fling Effect associated with the FP component.

The design ground motions developed shall represent the rock motions at depth (defined by the V_{S30} interface). These motions shall be used for the site-response analyses cited in Section 5.

Based on the results reported in the geotechnical and seismic hazard assessments, there are no known faults that cross the project site. The design of the tunnels shall therefore be based on the dual level earthquake ground motions, and it shall consider updated site-specific geotechnical and seismological data as part of the project-wide design requirements.

6.3 Tunnel Structures

Seismic demands on the tunnel structures and portals shall be determined by the modeling and analysis methods identified in Section 5 and the loading requirements specified in Section 6.2.

6.3.1 Service Load Effects

Effects of service loads such as geostatic pressures and dead load shall be used as the initial state of stress of the structure prior to seismic loading. These effects shall be combined with the seismic-induced displacement and loading effects.

6.3.2 Seismic Response Evaluation

The seismic response evaluations of the tunnels shall be conducted per the procedures stated in Section 5. The primary objective of these analyses shall be the determination of the rational distribution of deformations and loads on the soil-tunnel system.

6.3.3 Ductility Requirements & Performance Measures

The performance measures adopted for the cut-and-cover tunnels are based on the capacity of the of the tunnel cross sections to resist racking drift displacements in a ductile manner. The capacity analyses cited in Section 5 shall be used to establish the yield limit state and the corresponding ductility capacities of the structure, based on the strain limits specified in Section 6.3.4 for the service performance goals established by the FEE and SEE performance levels. The target ductilities for tunnel racking drift displacements shall be as follows: 2.0 for the FEE, and 4.0 for the SEE performance goals.

Reinforced concrete members with a flexural force D/C ratio (computed from elastic analysis) exceeding 1.5 shall be evaluated by means of a nonlinear strain-based performance method specified in Section 5 and shall comply with the material strain limits specified in Section 6.3.4. Two layers of reinforcement shall be used and sufficient cross-ties shall be provided to comply with the Caltrans confinement requirements in plastic hinge zones. The cross ties shall not be smaller than #4 bars and shall have minimum spacing of 6 inches vertically and 12 inches horizontally.

6.3.4 Allowable Concrete and Reinforcement Strain Limits

Capacities of reinforced concrete structural components and sub-systems shall be determined per deformation-based criteria and material strain limits, consistent with the Caltrans-SDC and Project-Specific Structure Design Criteria. Ultimate strain capacities and stress-strain relationships shall consider confining effects of cross-ties, per Caltrans-SDC, and shall be consistent in performance targets established by Caltrans for structure design of the bridges within the project.

For members designed to perform beyond the yield limit state (i.e., ductile members), the strains in the confined concrete core and reinforcement steel bars shall be limited to the specific limits stated in Tables 6.3 and 6.4.

Table 6.3 Confined Concrete Core Strain Limits

Performance Goal	Allowable Concrete Strain*
Serviceability performance	50% ε_{cu}
Reparability performance	67% ε_{cu}
Safety / No-Collapse performance	100% ε_{cu}

** ε_{cu}= ultimate concrete strain, per confinement effects and Caltrans requirements*
= 0.005 for the tunnel structures

Table 6.4 Reinforcement Steel Bar Strain Limits in Plastic Hinges

Performance Goal	Reinforcement Size	ε_{su} *	ε_{pg} **
Serviceability Performance	#10 Bars and Smaller	0.12	0.02
	Bars Larger Than #10	0.09	0.02
Reparability Performance	#10 Bars and Smaller	0.12	0.060
	Bars Larger Than #10	0.09	0.045
Safety / No-Collapse Performance	#10 Bars and Smaller	0.12	0.08
	Bars Larger Than #10	0.09	0.06

** ε_u = Ultimate reinforcing steel strain*
*** ε_{pg} = Allowable reinforcing steel strain*

6.3.5 Non-Ductile Actions

All structural members essential for the stability of the structural system of the tunnel structures shall have sufficient shear capacity to ensure maintaining gravity load carrying capacity. Shear capacities of all structural members shall be evaluated based on the Caltrans-SDC criteria.

6.3.6 Seismic Performance Evaluation

Based on the performance measures established above, the estimated seismic demand loads and deformations (D), and the strength-based (shear) and deformation-based (ductility) capacities (C), the seismic performance of the tunnel structural system shall be evaluated based on D/C ratios.

For compliance with the LRFD design approach, the load factors applicable to load effects (forces, moments, deformations) and component-specific resistance factors applicable the nominal resistance (i.e., capacities) shall be used to evaluate the Extreme-Ia and –Ib Limit States identified in Section 4.

6.4 Non-Standard Retaining Walls

Seismic demands on non-standard retaining walls shall be determined by the modeling and analysis methods identified in Section 5 and the loading requirements specified in Section 6.2.

6.4.1 Design Approach

The seismic design of the non-standard earth retaining walls shall be performed in accordance with the requirements of the AASGHTO LRFD Bridge Design Specifications, Section 11. The design shall account for site-specific ground motions and soil conditions, and interactions with the wall. The analyses shall be conducted as specified in Section 5.2. The displacement of the wall systems shall be evaluated and verified as to meeting the project performance objectives, and to ensure protection of adjacent buildings and facilities supported on the retained earth.

Upper bound displacement demands shall be quantified from the seismic analyses. Consistent with the project performance goals (such as serviceability, reparability, or no-collapse), material strain limits shall be established for the wall structures, with well detailed plastic hinge locations (if applicable to the wall system). Limits of deformations and movements established by local analytic models cited in Section 5.2 shall be used for the design.

The design shall also account for overall global stability of the wall structure, including resistance against sliding and overturning. The effects of inertia due to the mass of the wall shall be included in the seismic behavior and design of the wall, except that forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure for non-gravity cantilevered walls and anchored walls (as defined in AASHTO-LRFD Section 11).

7. OTHER DESIGN REQUIREMENTS

7.1 Serviceability Requirements

The cut-and-cover tunnel design shall comply with the following detailing requirements to ensure durability of the structures.

7.1.1 Exposure Conditions

The minimum concrete cover required to prevent exposure of reinforcement shall be as follows:

- 3.0" for concrete exposed to earth
- 2.0" for walls exposed to air
- 2.5" for upper surface of invert slabs

7.1.2 Deflections

Long term deflections of roof slabs shall be evaluated and maintained below limits specified by the requirements of ACI-318.

7.1.3 Crack Width and Control

Flexural crack widths shall be controlled through limiting peak stresses in tension zones of flexural members. The flexural crack width estimates for the tunnel ceiling (roof soffit) and invert slab surface shall be performed in accordance with the procedure of ACI 224R-01 'Control of Cracking in Concrete Structures'.

7.1.4 Detailing

Detailing of reinforcement shall be performed according to the requirements of ACI -315, and the durability requirements of ACI -201.2R.

7.1.5 Corrosion Protection

Corrosion protection design shall be consistent with Caltrans and project-specific criteria for a 75-year life in marine environment.

7.2 Fire Protection

Tunnel Fire Life Safety provisions shall primarily be governed by NFPA 502, as discussed separately in the project tunnel system criteria. In compliance with such requirements, when applicable, appropriate fire protection features shall be provided for the supporting structure components for the design level fire. Design verification under the damaged state of components due to fire conditions shall be checked and verified against project performance objectives as well as NFPA 502 requirements.

7.3 Water Proofing and Drainage Systems

Membrane waterproofing with geotextile backing shall be considered around the full perimeter of the tunnel structure. A durable waterproofing system shall be selected which can withstand full water head occurring at the tunnel sections, resist chemical attack from contaminates prevailing in the soils and groundwater conditions at each location, and limit water leakage into the tunnel to acceptable levels. Water shall be collected in designed drainage system consisting of the road and formation drains, allowing for regular inspection and maintenance.

Tunnel drainage systems shall be compatible with and meet Caltrans Bridge and Roadway Drainage design requirements.

7.3.1 Performance Requirements

The selected waterproofing system shall effectively eliminate water leakage into the completed tunnel structure.

Suitable non-metallic water stops, seals and sealants shall be provided at all joints or changes in the tunnel configuration to seal against water ingress. Concrete class, type, cover, and design crack width shall be designed compatible with the external waterproofing system to maintain water-tightness and durability of the system. Suitable measures including collection, channeling, toe drains, piping and sump pumps (subject to approval) may be provided to control infiltration flows to protect the tunnel finishes or tunnel structure from significant damage.

7.4 Buildings and Miscellaneous Structures

Buildings, miscellaneous structures, and non-structural components thereof shall be designed in accordance with load and resistance factor design, as permitted by the applicable material sections in the 2009 International Building Code (ICC IBC) [7], modifications by the 2007 California Building Code (CBC) [8], and recommended provisions for earthquake-resistant design of structures by Structural Engineers Association of California (SEAOC) [16].

7.5 Other Evaluations – Adjacent Structures

All new tunnel structures, fill areas, and earth retaining structures shall be designed to minimize ground movements from fill areas, tunneling, and excavation operations – including dewatering – to limit damage to adjacent buildings, structures, and the existing viaducts.

Settlements and ground movements resulting from changes in the state-of-stress within the ground mass supporting buildings and structures adjacent to an excavation shall be evaluated to assess potential risk to these structures and to identify buildings and structures requiring further investigation and/or protection.

The assessment of these settlements shall be developed in conjunction with the existing buildings/structures conditions (i.e. from field investigations or design plans if available), as well as site geotechnical conditions. The influence of the adjacent structure’s stiffness and configuration shall be considered in estimating these ground deformations and distortions.

8. REFERENCES

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